MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

CREAMERY BRIDGE
OVER ROLLINS BROOK
CHARLESTON, MAINE

STATE OF MAINE
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GEOTECHNICAL DESIGN SUMMARY

This report provides geotechnical recommendations for replacement of the Creamery Bridge over Rollins Brook in Charleston, Maine. The replacement structure will be a buried concrete arch founded on spread footings constructed on bedrock. The arch will incorporate head walls and prefabricated concrete modular walls flared away from the arch at an angle at all corners except the southwest corner which will begin parallel with the stream before turning south 90 degrees parallel to the highway. The design and construction recommendations below are discussed in greater detail in Section 7.0 Foundation Considerations and Recommendations.

Arch Stem Wall and Wingwalls – The arch stem walls, footings and wingwalls will be designed to resist all lateral earth loads, vehicular loads, superstructure loads, and any loads transferred through the superstructure. Arch stem walls, footings and wingwalls will be designed for all relevant strength, service and extreme limit states in accordance with AASHTO LRFD Bridge Design Specification 4th Edition, 2007, (herein referred to as LRFD).

The design of arch walls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of arch walls and wingwalls founded on spread footings on bedrock. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface. For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths (3/8ths) of the footing dimensions, in either direction.

Arch stem walls shall be designed as restrained, meaning that they are not free to rotate at the top, but rather will be pushed back into the adjacent fill in reaction to arch thrust. Earth loads shall be calculated using a passive earth pressure coefficient, K_p = 3.25, calculated using Rankine Theory for cantilever-type walls. The designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf. The contractor shall compact backfill in the "Critical Backfill Zone" in accordance with the manufacturer's recommendations but no less than 92% of the AASHTO T-180 maximum dry density. The contractor shall compact backfill and overfill in areas other than the Critical Backfill Zone in accordance with the MaineDOT Standard Specifications.

Earth loads on wingwalls shall be calculated using an active earth pressure coefficient, K_a, of 0.31 derived from Rankine Theory for cantilever wingwalls. The designer may assume backfill soil Type 4 properties as above. Additional lateral earth pressure due to construction or live load surcharge is required per Section 3.6.8 of the BDG for the arch stem walls and wingwalls (see section 7.2, Arch Stem Wall and Wingwall Design, in this report).

PCMG Retaining Walls – Design will require free-standing prefabricated concrete modular gravity (PCMG) retaining walls founded on a reinforced concrete footing on bedrock along the approach fills. The walls will be designed by a Professional Engineer subcontracted by the contractor as a design-build item. Design and construction of the walls should be in accordance with LRFD specifications and the MaineDOT Special Provision Section 635. The bearing resistance for the PCMG wall founded on a reinforced concrete footing on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 8.0 ksf for wall system bases 4 to 8 feet wide and 13.5 ksf for bases from 10 to 14 feet wide. Based on presumptive bearing resistance values, a factored bearing resistance of 16 ksf may be used to control settlement when analyzing the service limit state, and for preliminary footing sizing.

The bearing resistance for the bottom unit of the PCMG wall shall be checked for the extreme limit state with a resistance factor of 1.0. The overall stability of the wall system should be investigated at the Service I Load Combination with a resistance factor, φ, of 0.65. In general, spread footings at stream crossings should be founded a minimum of 2 feet below the calculated scour depth.

Failure by sliding shall be investigated by the wall subcontractor. A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of precast concrete wall segments founded on spread footings on bedrock. For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths ($3/8^{ths}$) of the footing dimensions, in either direction. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.36 (tan 20 degrees) at the foundation soil to concrete interfaces and a maximum frictional coefficient of 0.58 (tan 30 degrees) at the foundation soil to soil in-fill interfaces. For the lowest PCMG unit, the eccentricity of factored loads at the strength limit state shall not exceed three-eighths ($3/8^{ths}$) of the footing dimensions, in either direction.

Scour and Riprap – Bridge approach slopes and slopes at wingwalls should be armored with 3 feet of riprap in accordance with the MaineDOT Bridge Design Guide (BDG) Section 2.3.11. The riprap section shall be underlain by a Class A erosion control geotextile and a 1 foot thick layer of bedding material conforming to Item number 703.19, Granular Borrow for Underwater Backfill of the Standard Specification and as shown in Standard Detail 610(03). Riprap shall meet the requirements of Section 703.26, Plain and Hand Laid Riprap. Riprap shall extend 1.5 feet horizontally in front of walls before sloping down at a maximum 1.75H:1V slope to the existing ground surface. The toe of riprap sections shall be constructed 1 foot below the streambed elevation.

Factored Bearing Resistance for Spread Footings on Bedrock – The factored bearing resistance at the strength limit state for spread footings on bedrock should not exceed 8.0 ksf for footings 4 to 8 feet wide and 13.5 ksf for footings from 10 to 14 feet wide. Based on presumptive bearing resistance values, a factored bearing resistance of 16 ksf may be used when analyzing the service limit state and for preliminary footing sizing, as allowed in LRFD C10.6.2.6.1. In no instance shall the bearing stress exceed the nominal resistance of the footing concrete, which may be taken as $0.3f'c$. The minimum footing size is 2 feet wide regardless of the applied bearing pressure or bearing material.

Settlement – No grade rise is planned. Settlement beneath the new approaches will be negligible. Footings (leveling pads) constructed on compacted fill soils for any required walls may experience settlement on the order of ¼-inch or less. Differential settlements will also be on the order of ¼-inch or less. Most of the settlement will occur as the fill is placed and post construction settlement will be negligible. Settlement of arch stem wall footings or PCMG wall footings constructed on bedrock due to the elastic compression of the bedrock will be negligible and will occur during construction.

Frost Protection – Foundations placed on bedrock are not subject to heave by frost. Thus, there are no frost embedment requirements for project footings cast directly on sound bedrock. Retaining wall foundations placed on granular soils should be founded a minimum of 6.0 feet below finish exterior grade for frost protection. Riprap is not considered as contributing to the overall thickness of soils required for frost protection.

Seismic Design Considerations – In accordance with LRFD 3.10.1, seismic analysis is not required for buried concrete structures regardless of seismic zone.

Construction Considerations –

<u>Excavation</u>

- Construction of arch and retaining wall structures will require soil excavation. Earth support systems may be required.
- Remove the old abutments in their entirety.
- Prepare bedrock subgrade for arch footings by creating level benches or a completely level surface. Bedrock excavation may use conventional equipment, but may also require drilling and blasting methods. All loose bedrock and soil debris should be removed from bearing surfaces and the surfaces washed with high pressure water and air before concrete is placed for the arch foundations.

<u>Blasting</u>

- Where blasting is required, conduct pre and post-blast condition surveys, as well as, blast vibration monitoring at nearby residences and bridge structures in accordance with MaineDOT Standard Specification 105.2.6, Use of Explosives and industry standards at the time of blast.

<u>Dewatering</u>

- Control groundwater and surface water infiltration to permit construction in-the-dry.
- Cofferdams, temporary ditches, pumping from sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment may be needed to divert surface water or groundwater if significant seepage is encountered during excavation.

<u>Reuse of Excavated Soil and Bedrock</u>

- <u>Do not</u> use excavated existing subbase aggregate for pavement structure construction or to re-base shoulders or for abutment and wall backfill soil. Excavated subbase sand and gravel may be used as fill below subgrade elevation in fill embankment areas.
- <u>Do not</u> use excavated existing fill or glacial till soils for fill anywhere beneath the pavement structure, dressing slopes, abutments or walls. Use these soils to dress slopes only below the bottom elevation of the shoulder subbase gravel.
- Glacial till or existing fill soils may be used as common borrow in accordance with MaineDOT Standard Specification Sections 203 and 703. It may be necessary to spread out and dry portions of these soils that are excessively moist.

Embankment Fill Areas

- Bench existing fill slope soils in accordance with MaineDOT Standard Specification 203.09, Preparation of Embankment Area, where new fill slope extensions are constructed over existing slopes. Current plans are to construct 1.75:1 (H:V) riprap armored slopes where needed along the approach causeway.

Erosion Control

- Use MaineDOT Best Management Practices February 2008 to minimize erosion of fine-grained soils found on the project site.

1.0 INTRODUCTION

Creamery Bridge crosses Rollins Brook in Charleston, Maine, as shown on Sheet 1, Site Location Map found at the end of this report. The existing bridge was built in 1931 with a single-span concrete slab superstructure and concrete gravity abutments. The existing span length is approximately 14 feet. The bridge has experienced moderate to severe deterioration and the substructure concrete was extensively rehabilitated and widened in 1980. At present, there has been some section loss in the substructure abutments and the deck has undergone significant cracking and underside efflorescence. As of the year 2000, the bridge sufficiency rating was 52.1.

At the preliminary design report (PDR) meeting on 29 October 2008, the design team decided that the most practicable replacement for the existing bridge at this site is a buried concrete arch structure. The spread footings will be constructed directly on bedrock or seal concrete founded on bedrock. This alternative will result in an arch span length on the order of 17 feet.

2.0 GEOLOGIC SETTING

The Maine Geologic Survey "Surficial Geology of Dover-Foxcroft Quadrangle, Maine, Open-file No. 81-17" (1981) indicates that surficial soils in the vicinity of Creamery Bridge consist of glacial till. The site is also close to a swamp and tidal marsh deposit contact. The glacial till is typically a heterogeneous mixture of sand, silt, clay, and stones. The swamp deposit typically consists of peat, silt, clay and sand.

According to the Bedrock Geologic Map of Maine (1985), the bedrock at the Creamery Bridge site consists of Silurian age interbedded pelite and sandstone of the Waterville Formation.

3.0 SUBSURFACE INVESTIGATION

MaineDOT investigated subsurface conditions at the site by drilling two test borings in August 2008, BB-CBB-101 and BB-CBB-102. The approximate boring locations are shown on Sheet 2, Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report. We terminated both borings with bedrock cores. We present the details and sampling methods used, field data obtained, and soil and groundwater conditions encountered in the boring logs in Appendix A and on Sheet 3, Boring Logs, provided at the end of this report.

The MaineDOT drill crew used solid stem auger and cased wash boring techniques to conduct the borings. Soil samples were obtained, where possible, at 5-foot intervals using Standard Penetration Test (SPT) methods. The standard penetration resistances, or N-values, discussed in this report are corrected for average hammer energy transfer. We compute the corrected or, N_{60}-values, by applying an average hammer energy transfer factor of 0.77 to the raw field N-values obtained with the MaineDOT drill rig.

Bedrock was cored using an NQ-2 core barrel producing a 2.0-inch diameter rock core. The

MaineDOT geotechnical team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, and identified field and laboratory testing requirements. A MaineDOT Certified Subsurface Inspector logged the subsurface conditions encountered on the field logs. The MaineDOT survey crew determined the boring location coordinates in the field when they collected the project survey data.

4.0 LABORATORY TESTING

We conducted a laboratory soil testing program on selected samples recovered from the test borings to evaluate soil classification, material reuse, and subgrade soil properties. We also collected a stream bed sample for soil testing. Laboratory testing consisted of seven (7) standard grain size analyses with natural water content.

We present results of laboratory testing in Appendix B, Laboratory Test Data. The AASHTO and Unified Soil Classification System (USCS) soil classification and water content data are also presented on the boring logs in Appendix A.

5.0 SUBSURFACE CONDITIONS

Regional surficial geology maps show that the bridge site is located in an area of glacial till deposits. However, the bridge itself is situated at the end of short fill extensions built across the Rollins Brook flood plain. Consequently, the soil behind the abutments is predominantly granular fill and cobbles overlying approximately 6 feet of glacial till. We found the glacial till overlies bedrock. Both of the boring locations are underlain by phyllite bedrock. We provide an interpretive subsurface profile depicting the site stratigraphy on Sheet 2, Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report. A summary description of the subsurface conditions follows:

5.1 Granular Fill

We encountered granular fill to a depth of approximately 10.0 feet below ground surface (bgs) in BB-CBB-101, and to a depth of approximately 2.5 feet bgs in BB-CBB-102. Based on the boring logs, the fill layer is generally comprised of dense to very dense fine to coarse sand with little gravel and little to some silt with occasional cobbles. The SPT N_{60}-values in the granular fill ranged from 45 to 69 blows per foot (bpf) indicating that the unit is dense to very dense in consistency

The granular fill samples had water contents ranging between 4 and 6 percent. Grain size analyses conducted on selected samples of the fill soils indicate that the soils are classified as A-1-b by the AASHTO Classification System and SM under the Unified Soil Classification System.

5.2 Glacial Till

The glacial till found in the borings comprised fine to coarse sand with gravel to gravelly and little to some silt. The thickness of this soil unit ranged from approximately 5.5 feet in boring BB-CBB-101 to 7.0 feet in boring BB-CBB-102. SPT N_{60}-values ranged from 5 to 99 bpf, indicating these deposits are loose to very dense in consistency. We observed the glacial till unit over bedrock in each of the borings.

The glacial till samples had water contents ranging between 10 and 19 percent. Grain size analyses conducted on selected samples of the till soils indicate that the soils are classified as A-1-a, and A-2-4 by the AASHTO Classification System and SM and SW-SM under the Unified Soil Classification System.

5.3 Bedrock

We encountered bedrock at a depth of 15.0 feet bgs at BB-CBB-101 and 9.5 feet bgs at BB-CBB-102. Locally, the bedrock is mapped as the Waterville Formation which is made up of interbedded pelite and sandstone. Visual identification of rock cores indicates that the bedrock is a grey and orange, fine-grained, phyllite, soft and highly fractured, with 1 to 4 mm thick folded sandstone beds orthogonal to the phyllite cleavage. We determined that the rock quality designation (RQD) of the bedrock ranged from 0 to 25 percent which correlates to a very poor rock mass quality. The table below summarizes the top of bedrock elevations at the boring locations:

Substructure	Boring	Station	Depth to Bedrock (feet)	Elevation of Bedrock Surface (feet)
Abutment No. 1	BB-CBB-101	13+38.5, 11.3 LT	15.0	261.2
Abutment No. 2	BB-CBB-102	13+78.8, 5.2 RT	9.5	267.5

5.4 Groundwater

We observed groundwater at both of the boring locations. Groundwater occurred at a depth of 9.0 feet at BB-CBB-101 and 8.5 feet at BB-CBB-102. However, the groundwater level will fluctuate with seasonal changes, runoff, and adjacent construction activities.

For a more detailed description of the subsurface conditions, please refer to Appendix A, Boring Logs attached to this report.

6.0 FOUNDATION ALTERNATIVES

Site conditions and several replacement alternatives revolving around three-sided box concepts were discussed at the PDR meeting on 29 October 2008. The design team decided that the most practicable replacement for the existing bridge at this site is a buried concrete arch structure. The arch wall spread footings will be founded directly on bedrock or seal concrete founded on bedrock. This alternative will result in an arch span length on the order of 17 feet. Consequently, this report provides geotechnical design recommendations for arch walls, spread footing foundations and PCMG walls in Section 7.0 Foundation Considerations and Recommendations.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The design team has selected a buried concrete arch structure supported on cast-in-place spread footings constructed directly on bedrock or seal concrete to replace the bridge at the Charleston site. The design methodology used in the following evaluation is referenced from the AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

7.1 Spread Footings on Bedrock

The borings encountered bedrock approximately 9 to 15 feet below the existing bridge approaches at the boring locations. It is therefore considered feasible that cofferdams, seals (if required) and spread footings could be practically and economically constructed to bear on bedrock. The boring logs indicate that the bedrock at the site is highly fractured. Thus, it will be necessary to excavate all loose fractured or weathered bedrock before placing seal or spread footing concrete. Depending on the depth of the weathered bedrock excavation, a stem wall foundation may be required. The full extent of the rock excavation needed will not be known until the foundation excavation is made.

7.2 Arch Stem Wall and Wingwall Design

Arch stem walls and wingwalls shall be proportioned for all applicable load combinations in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength, service and extreme limit states. In addition to the typical loads, spread footing loads should consider all reactions transferred to the footings through the arch walls. The design of arch stem walls and wingwalls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. In accordance with LRFD Article 12.5.5., the resistance factor values for the geotechnical design of foundations for buried structures shall be as specified in LRFD Section 10, Foundations.

Extreme limit state design shall also consider foundation resistance after scour due to the design flood. In accordance with LRFD Article 10.5.5.3.2, the foundation resistance after scour shall be adequate to support the unfactored Strength Limit State loads with a resistance factor of 1.0.

A sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on bedrock. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface.

For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths ($3/8^{ths}$) of the footing dimensions, in either direction.

A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement, excessive horizontal movement and scour at the design flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65.

Wingwalls shall be designed as unrestrained, meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using an active earth pressure coefficient, K_a = 0.31, calculated using Rankine Theory for cantilever-type abutments and wingwalls. The designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Arch stem walls shall be designed as restrained, meaning that they are not free to rotate at the top, but rather will be pushed back into the adjacent fill in reaction to arch thrust. Earth loads shall be calculated using a passive earth pressure coefficient, K_p = 3.25, calculated using Rankine Theory for cantilever-type walls. The designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the BDG for the arch stem walls and wingwalls. The live load surcharge on wingwalls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet, per LRFD Table 3.11.6.4-2. The live load surcharge on arch stem walls may be estimated as a uniform earth pressure due to an equivalent height of soil (h_{eq}) taken from the table below:

Arch Stem Wall Height (feet)	h_{eq} (feet)
5.0	4.0
10.0	3.0
$\geq$ 20.0	2.0

Backfill within 10 feet of the arch walls and wingwalls and side slope fill shall conform to MaineDOT Specification 709.19, Granular Borrow for Underwater Backfill. This gradation specifies 10 percent or less of material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure. The contractor shall compact backfill in the “Critical Backfill Zone” in accordance with the manufacturer’s recommendations but no less than 92% of the AASHTO T-180 maximum dry density. The contractor shall compact backfill and overfill in areas other than the Critical Backfill Zone in accordance with the MaineDOT Standard Specifications.

Slopes in front of and sloping down to the wingwalls should be constructed with riprap and not exceed 1.75H:1V.

7.3 PCMG Retaining Walls

The selected foundation alternative will require prefabricated concrete modular gravity return walls. The walls shall be designed by a Professional Engineer subcontracted by the contractor as a design-build item. Design and construction of the walls should be in accordance with LRFD and the MaineDOT Special Provision Section 635. The wall shall also be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to 2.0 feet of soil.

The bearing resistance for the PCMG wall founded on a reinforced concrete footing on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 8.0 ksf for wall system bases 4 to 8 feet wide and 13.5 ksf for bases from 10 to 14 feet wide. The designer may assume the stress distribution to be a uniform distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-1. Based on presumptive bearing resistance values, a factored bearing resistance of 16 ksf may be used to control settlement when analyzing the service limit state, and for preliminary footing sizing. See Appendix C, Calculations, for supporting documentation.

The bearing resistance for the bottom unit of the PCMG wall shall be checked for the extreme limit state with a resistance factor of 1.0. The PCMG units must be designed so that the nominal bearing resistance, in conjunction with the depth of scour, provides adequate resistance to support the unfactored strength limit state loads with a resistance factor of 1.0. The overall stability of the wall system should be investigated at the Service I Load Combination with a resistance factor, φ, of 0.65. In general, spread footings at stream crossings should be founded a minimum of 2 feet below the calculated scour depth.

The designer shall apply a sliding resistance factor, φ_τ, of 0.80 to the nominal sliding resistance of precast concrete wall segments founded on spread footings on bedrock. For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths ($3/8^{ths}$) of the footing dimensions, in either direction. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.36 (tan 20 degrees) at the foundation soil to concrete interfaces and a maximum frictional coefficient of 0.58 (tan 30 degrees) at the foundation soil to soil in-fill interfaces. Recommended values of sliding frictional coefficients are based on LRFD Article 11.11.4.2, Table 10.5.5.2.2-1 and Table 3.11.5.3-1.

7.4 Scour and Riprap

The designer shall consider the consequences of changes in foundation conditions resulting from the design flood scour at the extreme and service limit states. The extreme limit evaluation for scour shall provide adequate foundation resistance to support the unfactored strength limit loads with a resistance factor of 1.0. These changes in foundation conditions shall be investigated at abutments, wingwalls and retaining walls.

In general, for scour protection, any footings for wingwalls or retaining walls which are constructed on soil should be embedded at least 2 feet below the design scour depth and armored with 3 feet of riprap for scour protection. Refer to BDG Section 2.3.11 for information regarding scour design.

The riprap layer shall be at least 3 feet thick. Stone riprap shall conform to MaineDOT Standard Specification 703.26, Plain and Hand Laid Riprap. For wingwalls and retaining walls, the riprap shall extend 1.5 feet horizontally in front of the walls before sloping at maximum 1.75H:1V slope to the existing ground surface. The toe of riprap sections shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a Class A erosion control geotextile and a 1 foot thick layer of bedding material conforming to MaineDOT Standard Specification 703.19, Granular Borrow for Underwater Backfill, as shown on Standard Detail 610 (03).

7.5 Factored Bearing Resistance

Concrete arch spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads are specified in LRFD Article 11.5.5. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2. The bearing resistance for any structure founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 8.0 ksf for footings 4 to 8 feet wide and 13.5 ksf for footings from 10 to 14 feet wide. This assumes a bearing resistance factor, φ_b, for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. The calculated factored bearing resistance is based on the natural fractured bedrock subgrade. A factored bearing resistance of 16 ksf may be used for preliminary footing sizing and to control settlements when analyzing the service limit state load combination. See Appendix C, Calculation, for supporting documentation.

In no instance shall the factored bearing stress exceed the factored compressive resistance of the footing concrete, which may be taken as $0.3f'c$. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

7.6 Settlement

The current bridge replacement plans do not include profile changes. No compressible soils or peat occur beneath the existing approach embankments. Consequently, settlement beneath approach embankments will be negligible.

Return walls may be necessary behind the abutments. We estimate that settlements beneath the wall leveling pads constructed on native soil or compacted granular fill will be on the order of ¼-inch. Differential settlement will also be on the order of ¼-inch or less. We anticipate that all of these settlements will occur during construction and will have minimal effect on the completed structure. We expect that any settlement of the arch footings will be due to the elastic compression of the bedrock. This settlement will be negligible and will occur during construction.

7.7 Frost Protection

We have evaluated the potential frost depth at the site. Based on State of Maine frost depth maps, BDG Figure 5-1, the site has a design-freezing index of approximately 1960 F-degree days. Considering an assumed water content of 15 to 20 percent, this correlates to a frost depth of approximately 6.0 feet. Consequently, we recommend that any foundations or leveling pads constructed at the site be founded a minimum of 6.0 feet below finished exterior grade. This minimum embedment applies only to foundations constructed on soil and not those founded on bedrock.

Abutment and return wing spread footings at the site will likely be founded on bedrock. Therefore, heave due to frost is not a design issue, and no requirements for minimum embedment depth are necessary.

7.8 Seismic Design Considerations

In accordance with LRFD Article 3.10.1, buried structures are not evaluated for seismic loading. Consequently, seismic earth loads do not need to be considered in arch substructure design.

7.9 Construction Considerations

7.9.1 Excavation

Construction of the new arch stem walls, spread footings and any retaining walls will require soil excavation. Earth support systems may be required.

We anticipate that the existing abutments will be removed entirely. Thus, a cofferdam may be required for construction of the new arch structure.

The arch wall foundation subgrade should consist of bedrock. The bearing surface should be cleaned of all overburden soils, and loose, disturbed, bedrock should be removed. We recommend final bedrock surface preparation by washing with a high pressure water jet.

The nature, slope, and degree of fracturing in the bedrock will not be evident until the foundation excavation is made. We recommend anchoring, doweling, or benching to create level steps as a means of improving sliding resistance if the prepared bedrock surface is steeper than 4:1 (H:V) in any direction.

Surface water should be diverted from the foundation excavation throughout the period of construction. We recommend removing any groundwater encountered at the base of the foundation excavation by using a sump pump located in a corner of the excavation outside of the foundation footprint.

The project may require PCMG retaining walls founded in native glacial till soils or compacted fill soils. The glacial till soil is susceptible to disturbance and rutting as a result of exposure to water or construction traffic. We recommend that the contractor protect the subgrade from exposure to water and any unnecessary construction traffic. If disturbance and rutting occur, we recommend that the contractor remove and replace the disturbed materials and replace with compacted gravel borrow. If the wall footing subgrade soil contains cobbles or boulders, we recommend that the contractor remove any cobbles and boulders larger than 6 inches in diameter.

If encountered, unsuitable soils should also be excavated from the footing subgrade to a depth of one foot and replaced with compacted gravel borrow prior to forming and casting the wall footings. Gravel borrow should conform to MaineDOT Standard Specification 703.20, Gravel Borrow. The gravel borrow should be compacted to 95 percent of the Modified Proctor maximum dry density (AASHTO T-180).

7.9.2 Blasting

Bedrock excavation may be needed to achieve level or benched wall subgrade elevation. The contractor should conduct all blasting work for the project in accordance with MaineDOT Standard Specification 105.2.6, Use of Explosives. We also recommend that the contractor conduct pre and post-blast surveys, as well as, blast vibration monitoring at nearby residences and bridge structures in accordance with industry standards at the time of blast.

7.9.3 Dewatering

The contractor should control groundwater and surface water infiltration to permit construction in-the-dry. We recommend that the contractor use coffer dams, temporary ditches, sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment to divert surface water and groundwater if significant seepage is encountered during construction. We also recommend using French drains daylighted to nearby ditches if significant seepage is encountered in the subgrade along the construction areas.

7.9.4 Reuse of Excavated Soil and Bedrock

The project plans call for excavation of the existing approach areas for bridge replacement. In the process, the contractor will excavate both the existing subbase gravel, and subgrade fill soils. We do not recommend using the excavated subbase aggregate to re-base the bridge approaches. Excavated subbase and any granular fill excavation may be used as fill below subgrade elevation in fill embankment areas provided all other requirements of MaineDOT Standard Specification Sections 203 and 703 are met.

We do not recommend using excavated glacial till soils as fill directly beneath the pavement structure. The glacial till is typically susceptible to strength loss when wet or disturbed. The excavated till soils may be allowed as fill in accordance with the Standard Specification 203 as shown on Standard Detail 203 (01). This soil may also be used for dressing slopes, but only below the bottom elevation of the shoulder subbase gravel.

The native glacial till or existing fill soils may be used as common borrow in accordance with MaineDOT Standard Specification Sections 203 and 703. Contractors should expect that prior to placement and compaction it may be necessary to spread out and dry portions of these soils that are excessively moist.

7.9.5 Embankment Fill Areas Outside of Arch/Wingwall Backfill Envelope

Embankment approach slopes that are created or extended as part of the bridge construction effort should be designed as earth fill slopes no steeper than 2:1 (H:V). Slopes steeper than 2:1 (H:V) typically require reinforcement or rock fill surfacing. Current plans are to construct 1.75:1 (H:V) riprap armored slopes where needed along the approach causeway.

We recommend that all new embankment fill be thoroughly and systematically compacted to the full limit of the slope. Where new fill slope extensions are constructed over existing slopes, we recommend benching the existing slope soils in accordance with MaineDOT Standard Specification 203.09, Preparation of Embankment Area, to prevent creation of a preferential slip plane under the new embankment fill.

The new embankment fill loads and densification of the fill materials during construction will result in ground surface settlement and consolidation of the underlying soils. We anticipate that most of this settlement will occur during and immediately after construction of the embankments. Post-construction settlement is expected to be minimal.

7.9.6 Erosion Control Recommendations

The fine-grained soils along the project are susceptible to erosion. We recommend using appropriate erosion control measures during construction as described in the MaineDOT Best Management Practices February 2008 guidelines to minimize erosion of the fine-grained soils at the site.

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for specific application to the replacement of the Creamery Bridge over Rollins Brook in Charleston, Maine. We have prepared the report in accordance with generally accepted soil and foundation engineering practices. No other intended use or warranty is expressed or implied.

In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations completed at discrete locations on the project site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

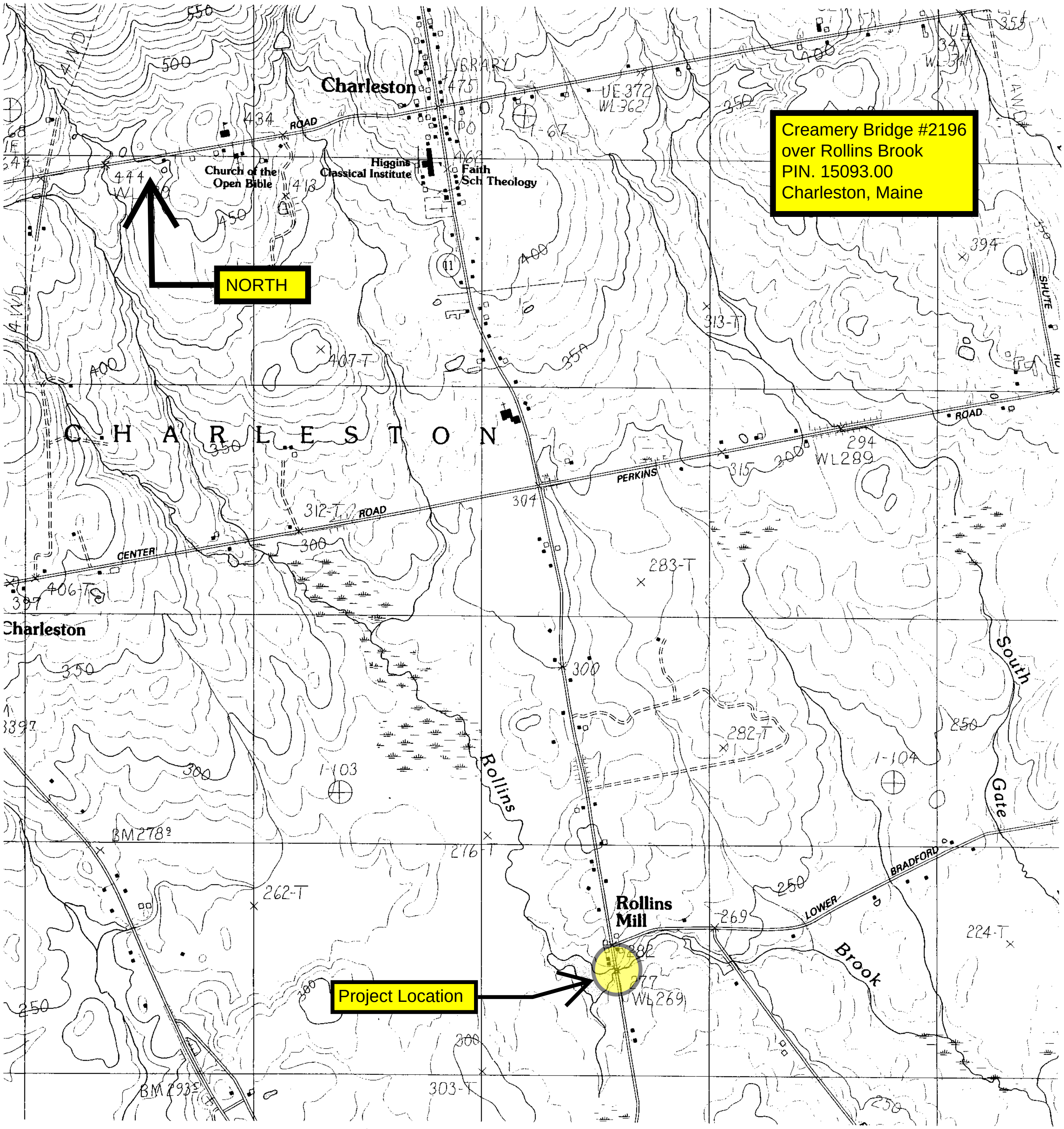
We recommend that we be provided the opportunity for a general review of the final design drawings and specifications in order that we may verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

REFERENCES

AASHTO, (2007), AASHTO LRFD Bridge Design Specifications, Fourth Edition, 2007, AASHTO, Washington, D.C.

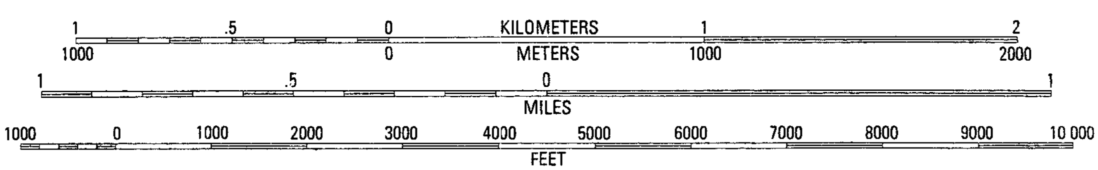
MaineDOT, (2003), Bridge Design Guide, MaineDOT Bridge Program, Augusta, ME.

Sheets



CHARLESTON QUADRANGLE
MAINE
7.5 MINUTE SERIES (TOPOGRAPHIC)

SCALE 1:24 000



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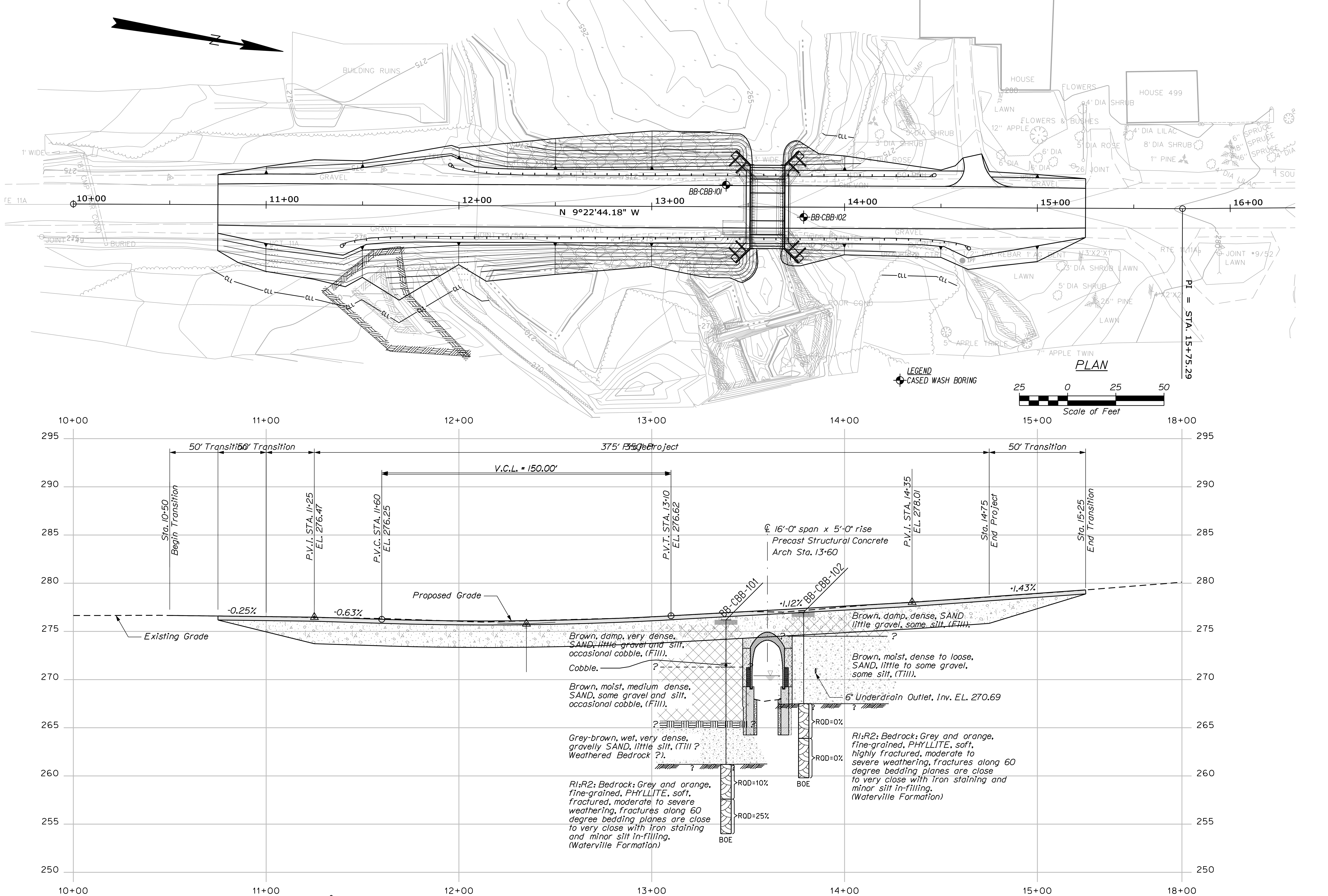

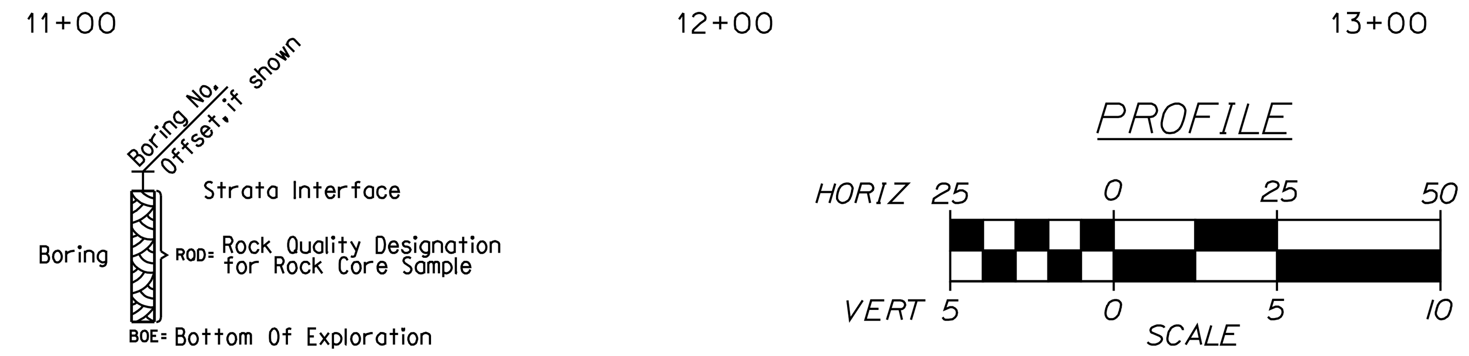

Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

15093.00

PIN 15093.00

BRIDGE NO. 2196

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	T. WHITE	OCT 2008
CHECKED-REVIEWED	M. MOREAU	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

CREAMERY BRIDGE
ROLLINS BROOK
CHARLESTON PENOBSCOT COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\msta\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:1/27/2009

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Creamery Bridge #2196 over Rollins Brook on Route 11
Location: Charleston, Maine

Boring No.: BB-CBB-101
PIN: 15093.00

Driller:	MaineDOT	Elevation (ft.)	276.2	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/14/08; 12:00-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	13+38.5, 11.3 Lt.	Casing ID/OD:	NW	Water Level*:	9.0' bgs.
Hammer Efficiency Factor: 0.77		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	275.70	PAVEMENT. 0.50	
	1D	24/18	1.00 - 3.00	8/20/34/23	54	69			Brown, damp, very dense, fine to coarse SAND, little gravel and silt, occasional cobble. (Fill)	G#208677 A-1-b, SM WC=4.9%
								271.30	Cobble from 4.5-4.9' bgs. 4.90	
5	2D	24/9	5.00 - 7.00	8/9/12/26	21	27			Brown, moist, medium dense, fine to coarse SAND, some gravel and silt, occasional cobble. (Fill)	G#208678 A-1-b, SM WC=4.0%
							9		Roller Coned ahead from 8.0-10.0' bgs. Wood fragment from 8.4-8.6' bgs.	
							19			
10	3D	24/20	10.00 - 12.00	10/35/42/37	77	99	36	265.70	10.50 Grey-brown, wet, very dense, gravelly fine to coarse SAND, little silt. (Till? Weathered Bedrock?)	G#208679 A-1-a, SW-SM WC=11.0%
							57			
							78		Roller Coned ahead from 13.0-15.0' bgs.	
							42			
15	R1	43.2/ 43.2	15.00 - 18.60	RQD = 10%			NQ-2	261.20	15.00 Bedrock: Grey and orange, fine-grained, PHYLLITE, soft, fractured, with 1 mm to 4 mm folded sandstone beds orthogal to the phyllite cleavage. Moderate to severe weathering, fractures close to very close along bedding planes primarily 60 degrees from horizontal, some fractures occur horizontal to vertical. Fractures are generally tight, but several are open with iron staining and minor silt in-filling. [Waterville Formation] R1: Core Times (min:sec) 15.0-16.0' (3:07) 16.0-17.0' (3:54) 17.0-18.0' (4:20) 18.0-18.6' (5:00) 100% Recovery, Core Blocked	
	R2	43.2/ 43.2	18.60 - 22.20	RQD = 25%				257.60	18.60 R2: Core Times (min:sec) 18.6-19.6' (4:20) 19.6-20.6' (5:06) 20.6-21.6' (3:16) 21.6-22.2' (no time recorded) 100% Recovery, Core Blocked	
20								254.00	22.20 Bottom of Exploration at 22.20 feet below ground surface.	
25										

Remarks:

100-150# down pressure on bit. No water return.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1

Boring No.: BB-CBB-101

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Creamery Bridge #2196 over Rollins Brook on Route 11
Location: Charleston, Maine

Boring No.: BB-CBB-102
PIN: 15093.00

Driller:	MaineDOT	Elevation (ft.)	277.0	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/14/08	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	13+78.8, 5.2 Rt.	Casing ID/OD:	NW	Water Level*:	8.5' bgs.
Hammer Efficiency Factor: 0.77		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
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N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	276.50	PAVEMENT. 0.50	
	1D/A	24/17	1.00 - 3.00	15/22/13/9	35	45			(1D) 1.0-2.5' bgs. Brown, damp, dense, fine to coarse SAND, little gravel, some silt. (Fill)	G#208680 A-1-b, SM WC=6.1%
								274.50	2.50 (1D/A) 2.5-3.0' bgs. Brown, moist, dense, fine to coarse SAND, little gravel, some silt, (Till).	G#208681 A-2-4, SM WC=10.4%
5	2D	24/23	5.00 - 7.00	3/2/2/2	4	5	6		Brown, wet, loose, fine to coarse SAND, some gravel and silt, (Till).	G#208682 A-2-4, SM WC=19.1%
							18			
							33			
							18		a40 blows for 0.5'.	
10	R1	43.2/ 40.8	9.50 - 13.10	RQD = 0%			a40 NQ-2	267.50	9.50 Bedrock: Grey and orange, fine-grained, PHYLLITE; soft, highly fractured, with 1 mm to 4 mm folded sandstone beds orthogal to the phyllite cleavage. Moderate to severe weathering, fractures close to very close along bedding planes primarily 60 degrees from horizontal, some fractures occur horizontal to vertical. Fractures are generally tight, but several are open with iron staining and minor silt in-filling. [Waterville Formation] R1: Core Times (min:sec) 9.5-10.5' (5:50) 10.5-11.5' (6:03) 11.5-12.5' (6:00) 12.5-13.1' (5:00) 94% Recovery Core Blocked	
	R2	49.2/ 49.2	13.10 - 17.20	RQD = 0%				263.90	13.10 R2: Core Times (min:sec) 13.1-14.1' (6:22) 14.1-15.1' (7:02) 15.1-16.1' (7:07) 16.1-17.2' (4:00) 100% Recovery Core Blocked	
15								259.80	17.20 Bottom of Exploration at 17.20 feet below ground surface.	
20										
25										

Remarks:

100-150# down pressure on bit.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1

Boring No.: BB-CBB-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
15093.00
PIN 15093.00
BRIDGE NO. 2196
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	M. MOREAU	
CHECKED-REVIEWED	T. WHITE	OCT 2008
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

CREAMERY BRIDGE
ROLLINS BROOK
CHARLESTON PENOBSCOT COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Creamery Bridge #2196 over Rollins Brook on Route 11
Location: Charleston, Maine

Boring No.: BB-CBB-101
PIN: 15093.00

Driller:	MaineDOT	**Elevation (ft.)**	276.2	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/14/08; 12:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+38.5, 11.3 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	9.0' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
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SSA = Solid Stem Auger
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RC = Roller Cone
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WOR/C = weight of rods or casing
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T_v = Pocket Torvane Shear Strength (psf)
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	275.70		PAVEMENT. 0.50	
	1D	24/18	1.00 - 3.00	8/20/34/23	54	69				Brown, damp, very dense, fine to coarse SAND, little gravel and silt, occasional cobble. (Fill)	G#208677 A-1-b, SM WC=4.9%
										Cobble from 4.5-4.9' bgs. 4.90	
5	2D	24/9	5.00 - 7.00	8/9/12/26	21	27		271.30		Brown, moist, medium dense, fine to coarse SAND, some gravel and silt, occasional cobble. (Fill)	G#208678 A-1-b, SM WC=4.0%
							9			Roller Coned ahead from 8.0-10.0' bgs. Wood fragment from 8.4-8.6' bgs.	
							19				
10	3D	24/20	10.00 - 12.00	10/35/42/37	77	99	36	265.70		10.50 Grey-brown, wet, very dense, gravelly fine to coarse SAND, little silt. (Till? Weathered Bedrock?)	G#208679 A-1-a, SW-SM WC=11.0%
							57				
							78				
							42			Roller Coned ahead from 13.0-15.0' bgs.	
15	R1	43.2/43.2	15.00 - 18.60	RQD = 10%			NQ-2	261.20		15.00 Bedrock: Grey and orange, fine-grained, PHYLLITE, soft, fractured, with 1 mm to 4 mm folded sandstone beds orthogal to the phyllite cleavage. Moderate to severe weathering, fractures close to very close along bedding planes primarily 60 degrees from horizontal, some fractures occur horizontal to vertical. Fractures are generally tight, but several are open with iron staining and minor silt in-filling. [Waterville Formation]	
	R2	43.2/43.2	18.60 - 22.20	RQD = 25%				257.60		R1: Core Times (min:sec) 15.0-16.0' (3:07) 16.0-17.0' (3:54) 17.0-18.0' (4:20) 18.0-18.6' (5:00) 100% Recovery, Core Blocked	
										18.60	
20										R2: Core Times (min:sec) 18.6-19.6' (4:20) 19.6-20.6' (5:06)	
								254.00		20.6-21.6' (3:16) 21.6-22.2' (no time recorded) 100% Recovery, Core Blocked	
										22.20 **Bottom of Exploration at 22.20 feet below ground surface.**	
25											

Remarks:

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Boring No.: BB-CBB-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Creamery Bridge #2196 over Rollins Brook on Route 11
Location: Charleston, Maine

Boring No.: BB-CBB-102
PIN: 15093.00

Driller:	MaineDOT	**Elevation (ft.)**	277.0	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/14/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+78.8, 5.2 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	8.5' bgs.
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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	1D/A	24/17	1.00 - 3.00	15/22/13/9	35	45				(1D) 1.0-2.5' bgs. Brown, damp, dense, fine to coarse SAND, little gravel, some silt. (Fill)	G#208680 A-1-b, SM WC=6.1%
								274.50		2.50 (1D/A) 2.5-3.0' bgs. Brown, moist, dense, fine to coarse SAND, little gravel, some silt, (Till).	G#208681 A-2-4, SM WC=10.4%
5	2D	24/23	5.00 - 7.00	3/2/2/2	4	5	6			Brown, wet, loose, fine to coarse SAND, some gravel and silt, (Till).	G#208682 A-2-4, SM WC=19.1%
							18				
							33				
							18			a40 blows for 0.5'.	
	R1	43.2/40.8	9.50 - 13.10	RQD = 0%			a40 NQ-2	267.50		9.50 Bedrock: Grey and orange, fine-grained, PHYLLITE; soft, highly fractured, with 1 mm to 4 mm folded sandstone beds orthogal to the phyllite cleavage. Moderate to severe weathering, fractures close to very close along bedding planes primarily 60 degrees from horizontal, some fractures occur horizontal to vertical. Fractures are generally tight, but several are open with iron staining and minor silt in-filling. [Waterville Formation]	
10											
								263.90		R1: Core Times (min:sec) 9.5-10.5' (5:50) 10.5-11.5' (6:03) 11.5-12.5' (6:00) 12.5-13.1' (5:00) 94% Recovery Core Blocked	
	R2	49.2/49.2	13.10 - 17.20	RQD = 0%						13.10	
15								259.80		R2: Core Times (min:sec) 13.1-14.1' (6:22) 14.1-15.1' (7:02) 15.1-16.1' (7:07) 16.1-17.2' (4:00) 100% Recovery Core Blocked 17.20	
20										**Bottom of Exploration at 17.20 feet below ground surface.**	
25											

Remarks:

100-150# down pressure on bit.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Boring No.: BB-CBB-102

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

Desired Soil Observations: (in this order)
Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Consistency is rated according to standard penetration resistance.

Modified Burmister System

Descriptive Term	Portion of Total
trace	0% - 10%
little	11% - 20%
some	21% - 35%
adjective (e.g. sandy, clayey)	36% - 50%

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value blows per foot	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily Penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

*Minimum NQ rock core (1.88 in. OD of core)

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	RQD
Very Poor	<25%
Poor	26% - 50%
Fair	51% - 75%
Good	76% - 90%
Excellent	91% - 100%

Desired Rock Observations: (in this order)
Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

Sample Container Labeling Requirements:

PIN	Blow Counts
Bridge Name / Town	Sample Recovery
Boring Number	Date
Sample Number	Personnel Initials
Sample Depth	

Appendix B

Laboratory Test Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Charleston Project Number: 15093.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-CBB-101, 1D	13+38.5	11.3 Lt.	1.0-3.0	208677	1	4.9			SM	A-1-b	II
BB-CBB-101, 2D	13+38.5	11.3 Lt.	5.0-7.0	208678	1	4.0			SM	A-1-b	II
BB-CBB-101, 3D	13+38.5	11.3 Lt.	10.0-12.0	208679	1	11.0			SW-SM	A-1-a	0
BB-CBB-102, 1D	13+78.8	5.2 Rt.	1.0-2.5	208680	1	6.1			SM	A-1-b	II
BB-CBB-102, 1D/A	13+78.8	5.2 Rt.	2.5-3.0	208681	1	10.4			SM	A-2-4	II
BB-CBB-102, 2D	13+78.8	5.2 Rt.	5.0-7.0	208682	1	19.1			SM	A-2-4	II
RIVER BED SAMPLE				211452	2	17.4			SW	A-1-a	0

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

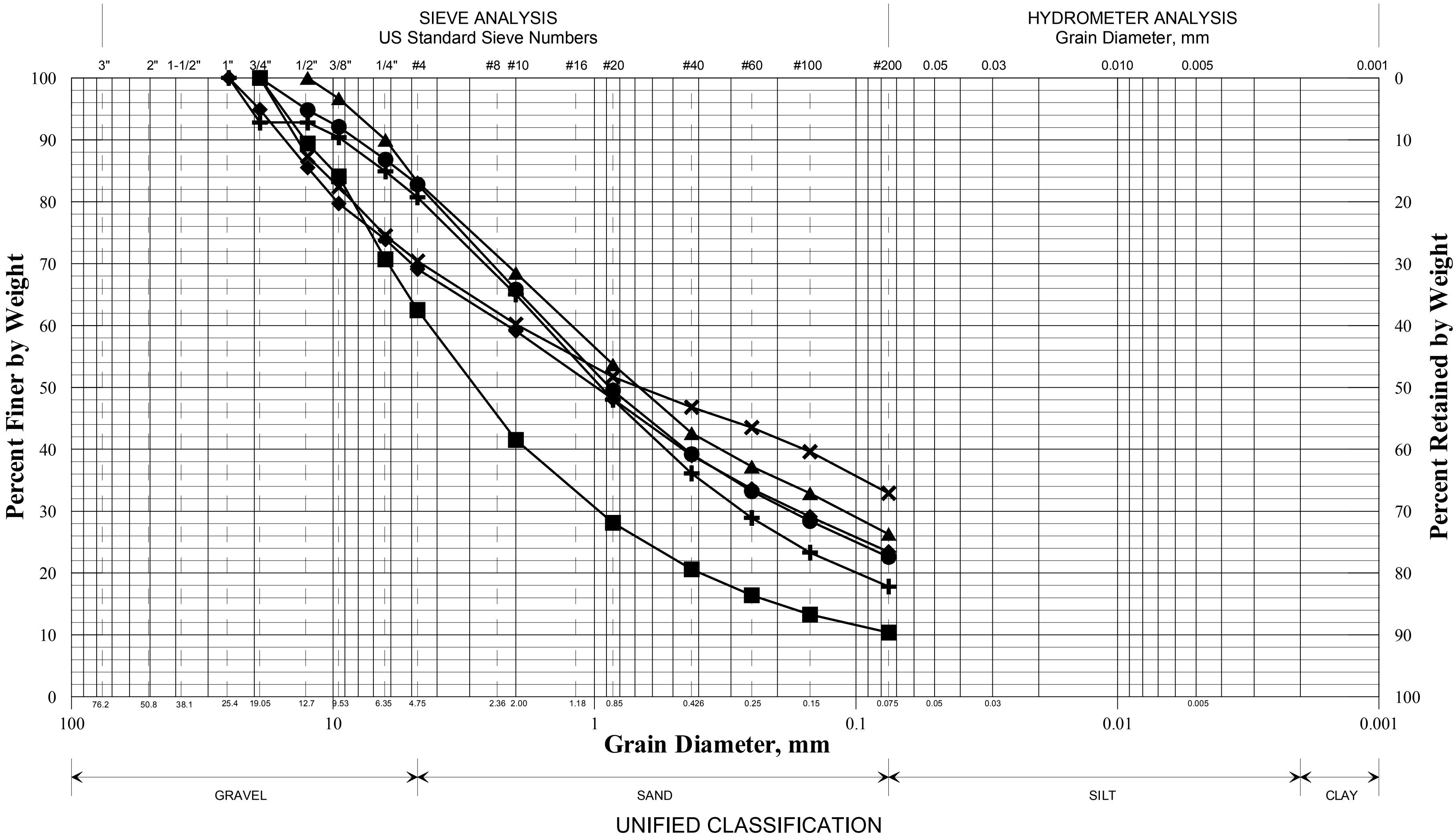
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

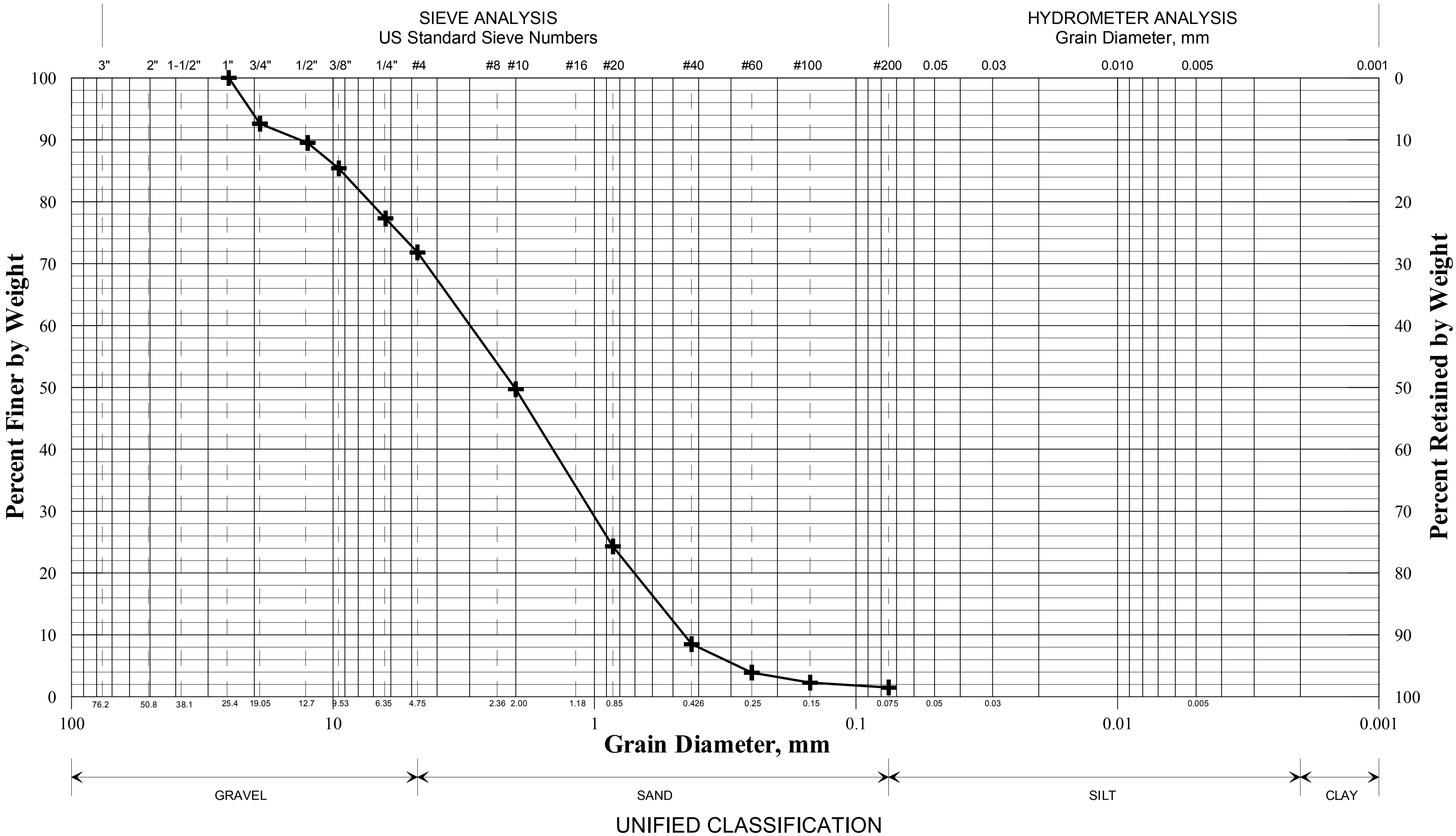


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-CBB-101/1D	13+38.5	11.3 LT	1.0-3.0	SAND, little gravel, little silt.	4.9			
◆	BB-CBB-101/2D	13+38.5	11.3 LT	5.0-7.0	SAND, some gravel, some silt.	4.0			
■	BB-CBB-101/3D	13+38.5	11.3 LT	10.0-12.0	Gravelly SAND, little silt.	11.0			
●	BB-CBB-102/1D	13+78.8	5.2 RT	1.0-2.5	SAND, some silt, little gravel.	6.1			
▲	BB-CBB-102/1D(A)	13+78.8	5.2 RT	2.5-3.0	SAND, some silt, little gravel.	10.4			
✖	BB-CBB-102/2D	13+78.8	5.2 RT	5.0-7.0	SAND, some silt, some gravel.	19.1			

PIN
015093.00
Town
Charleston
Reported by/Date
WHITE, TERRY A 10/2/2008

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	RIVER BED SAMPLE				SAND, some gravel, trace silt.	17.4			
◆									
■									
●									
▲									
✖									

PIN
015093.00
Town
Charleston
Reported by/Date
WHITE, TERRY A 11/20/2008

SHEET 2

Appendix C

Calculations

ABUTMENT AND WINGWALL ACTIVE AND PASSIVE EARTH PRESSURES:

Rankine Theory - Active Earth Pressure from MaineDOT Bridge Design Guide Section 3.6.5.2, pg. 3-7

Either Rankine or Coulomb may be used for long-heeled cantilever walls where the failure surface is uninterrupted by the top of the wall stem. In general, use Rankine though.

Soil angle of internal friction: $\phi := 32deg$

Slope angle of backfill soil from horizontal: $\beta := 0deg$

$$K_a := \tan\left[45deg - \left(\frac{\phi}{2}\right)\right]^2$$

$K_a = 0.31$

Rankine Theory - Passive Earth Pressure from Bowles 5th Edition Section 11-5, pg 602

Soil angle of internal friction: $\phi := 32deg$

Slope angle of backfill soil from horizontal: $\beta := 0deg$

$$K_{p\_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

$K_{p\_rank} = 3.25$

Coulomb Theory - Active Earth Pressure from MaineDOT Bridge Design Guide Section 3.6.5.2, pg. 3-7

For gravity walls , semi-gravity walls, prefabricated modular walls, and cantilever walls and abutments with short heels where wall and backfill interface friction is considered, use Coulomb Theory

Angle of back face of wall: $\alpha := 90deg$

Soil angle of internal friction: $\phi := 32deg$

Slope angle of backfill soil from horizontal: $\beta := 0deg$

$\delta = \beta$ $\delta := \beta$

$$K_a := \frac{\sin(\alpha + \phi)^2}{\sin(\alpha)^2 \cdot \sin(\alpha - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\sin(\alpha - \delta) \cdot \sin(\beta + \alpha)}}\right)^2}$$

$K_a = 0.31$

Coulomb Theory - Passive Earth Pressure from MaineDOT Bridge Design Guide
Section 3.6.6, pg. 3-8

Angle of back face of wall: $\alpha := 90\text{deg}$

Soil angle of internal friction: $\phi := 32\text{deg}$

Friction angle between fill and wall:
From LRFD Table 3.11.5.3-1, pg. 3-74, δ ranges from 17 to 22 $\delta := 20\text{deg}$

Angle of backfill from horizontal: $\beta := 0\text{deg}$

$$K_p := \frac{\sin(\alpha-\phi)^2}{\sin(\alpha)^2\cdot\sin(\alpha+\delta)\cdot\left(1-\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\beta)}{\sin(\alpha-\delta)\cdot\sin(\beta+\alpha)}}\right)^2}$$

$K_p = 6.89$

Frost Protection:

From the Maine Design Freezing Index Map:

DFI = 1960 degree-days

Site has Granular Soils With Wn = 15% to 20%

From the 2003 Bridge Design Guide Table 5-1:

$\text{Frost\_depth} := [0.6\cdot(78.7\text{in}-76.6\text{in})+76.6\text{in}]$

$\text{Frost\_depth} = 77.86\cdot\text{in}$

$\text{Frost\_depth} = 6.49\cdot\text{ft}$

Use 6.0 feet

BEARING RESISTANCE - FOOTINGS ON COMPACTED FILL SOILS:

Consider this for use with PCMG and Wingwalls; however it's likely that all footings will bear on bedrock.

SERVICE LIMIT STATE:

LRFD Table C10.6.2.6.1-1, (Based on NAVFAC DM 7.2) - "Presumptive Bearing Resistances for Spread Footing Foundations at the Service Limit State"

Bearing Material	Consistency in Place	Bearing Resistance (kips per sq. foot)	Recommended Value
Coarse to Medium sand, little gravel	Very dense	8 to 12	8 ksf
	Medium dense to dense	4 to 8	6 ksf
	Loose	2 to 4	3 ksf

Recommend **6.0 ksf** to control settlements for **Service Limit State** analyses and for preliminary footing sizing.

STRENGTH LIMIT STATE:

Nominal and Factored Bearing Resistance for spread footings on fill soils at the Strength Limit State:

Assumptions:

1. Footings will be embedded 6.0 feet for frost protection.

$D_f := 6.0ft$

2. Assumed parameters for soils:
   Assume granular fill

Moist unit weight: $\gamma_m := 125pcf$

Saturated unit weight: $\gamma_{sat} := 130pcf$

Soil angle of internal friction: $\phi_{ns} := 32$

Undrained shear strength (cohesion): $c_{ns} := 0psf$

3. Use Terzaghi strip equations as L > B

Depth to Groundwater table based on boring data: $D_w := 0 \cdot ft$

Unit weight of water: $\gamma_w := 62.4pcf$

Effective Stress at the footing bearing level: $q_{eff\_str} := D_w \cdot \gamma_m + (D_f - D_w) \cdot (\gamma_{sat} - \gamma_w)$

$q_{eff\_str} = 0.41 \cdot ksf$

Look at several footing widths:

$$B := \begin{pmatrix} 4 \\ 6 \\ 8 \\ 10 \\ 12 \\ 14 \end{pmatrix} ft$$

Terzaghi Shape Factors from Table 4-1, p. 220
For strip footing:

$s_c := 1.0$

$s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors For ϕ = 32 deg   Bowles 5th Ed. Table 4-4 pg. 223

$N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 22.0$

Nominal Bearing Resistance per Terzaghi equation   Bowles 5th Ed. Table 4-1 pg. 220

$$q_{nom} := c_{ns} \cdot N_c \cdot s_c + q_{eff\_str} \cdot N_q + 0.5(\gamma_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_{nom} = \begin{pmatrix} 12.4 \\ 13.9 \\ 15.4 \\ 16.8 \\ 18.3 \\ 19.8 \end{pmatrix} \cdot ksf$$

Resistance Factor from LRFD Table 10.5.5.2.2-1 pg. 10-32: $\phi_b := 0.45$

$$q_{fac} := q_{nom} \cdot \phi_b$$

$$q_{fac} = \begin{pmatrix} 5.6 \\ 6.2 \\ 6.9 \\ 7.6 \\ 8.2 \\ 8.9 \end{pmatrix} \cdot ksf$$

Recommend **Strength Limit State** Factored Bearing Resistance of **5.5 ksf** for wall bases and footings 4 to 8 feet wide and **7.5 ksf** for wall bases and footings 10 to 14 feet wide.

BEARING RESISTANCE - FOOTINGS ON BEDROCK:

SERVICE LIMIT STATE:

Method 1

Method: , based on
LRFD Table C10.6.2.6.1-1, (Based on NAVFAC DM 7.2) - "Presumptive Bearing Resistances for Spread Footing Foundations at the Service Limit State"

Description of Bedrock Materials:

Boring BB-CBB-101: Highly fractured PHYLLITE, RQD 10-25%

Boring BB-CBB-102: Highly fractured PHYLLITE, RQD 0%

Bearing Material:	Weathered bedrock, RQD less than 25%
Consistency in Place:	Medium hard rock
Bearing Resistance:	Range 16-24 ksf
Recommended Value	16 ksf

Use a Factored Bearing Resistance of **16 ksf** for **Service Limit State** analysis and preliminary sizing of the footings.

Method 2

Method: AASHTO Standard Specifications - 17th Edition, 2002

Section 4.4.8.1.1 - Competent Rock
Figure 4.4.8.1.1A - for footings supported on competent rock
Average RQD of site bedrock is 0%

Allowable contact stress: 10 tsf (20 ksf)

STRENGTH LIMIT STATE:

Method 3

Method: AASHTO Standard Specifications - 17th Edition, 2002

Section 4.4.8.1.2 - Footings on Broken or Jointed Rock Competent Rock
Figure 4.4.8.1.1A - for footings supported on jointed rock

a. estimated Rock Mass Rating — Very Poor (RQD ~0)

b. Rock Category per 4.4.8.1.2B — B, Phyllite

c. Unconfined compressive strength, Co — 3500 - 35000 psi

d. N_{ms}, per Table 4.4.8.1.2A — Use q_{ult} of equivalent soil mass

e. $Q_{ult} = Q_{nom}$ — q_{ult} of equivalent soil mass

Nominal Bearing Resistance for Spread Footings on Fractured Bedrock Using Equivalent Soil Mass:

Use Terzaghi Strip Footing Equation to Calculate Q_{nom}.

Assumptions:

1. Footings will be embedded 7.5 feet for frost protection.

   $D_f := 3.0ft$ — Assume only Riprap Layer Burial

2. Assumed parameters for soils:
   Assume granular fill

   Moist unit weight: $\gamma_m := 145pcf$

   Saturated unit weight: $\gamma_{sat} := 150pcf$

   Soil angle of internal friction: $\phi_{ns} := 36$ — Assume similar to dense till

   Undrained shear strength (cohesion): $c_{ns} := 0psf$

3. Use Terzaghi strip equations as L > B

Depth to Groundwater table based on boring data: $D_w := 0 \cdot ft$

Unit weight of water: $\gamma_w := 62.4pcf$

Effective Stress at the footing bearing level: $q_{eff\_str} := D_w \cdot \gamma_m + (D_f - D_w) \cdot (\gamma_{sat} - \gamma_w)$

$q_{eff\_str} = 0.26 \cdot ksf$

Look at several footing widths:

$$B := \begin{pmatrix} 4 \\ 6 \\ 8 \\ 10 \\ 12 \\ 14 \end{pmatrix} \cdot ft$$

Terzaghi Shape Factors from Table 4-1, p. 220
For strip footing:

$s_c := 1.0$

$s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors For ϕ = 36 deg

Bowles 5th Ed. Table 4-4 pg. 223

$N_c := 50.55$ $\quad N_q := 37.7$ $\quad N_\gamma := 44.4$

Nominal Bearing Resistance per Terzaghi equation

Bowles 5th Ed. Table 4-1 pg. 220

$$Q_{nom} := c_{ns} \cdot N_c \cdot s_c + q_{eff\_str} \cdot N_q + 0.5 \left(\gamma_{sat} - \gamma_w\right) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$\left(Q_{nom}\right) = \begin{pmatrix} 17.7 \\ 21.6 \\ 25.5 \\ 29.4 \\ 33.2 \\ 37.1 \end{pmatrix} \cdot ksf$$

Resistance Factor from LRFD Table 10.5.5.2.2-1 pg. 10-32:

$\phi_b := 0.45$

$$q_{fac} := Q_{nom} \cdot \phi_b$$

Factored Bearing Resistance

$$q_{fac} = \begin{pmatrix} 8 \\ 9.7 \\ 11.5 \\ 13.2 \\ 15 \\ 16.7 \end{pmatrix} \cdot ksf$$

Recommend **Strength Limit State** Factored Bearing Resistances of **8 ksf** for wall bases and footings 4 to 8 feet wide and **13.5 ksf** for wall bases and footings 10 to 14 feet wide.

SETTLEMENT ANALYSIS:

Estimate Settlement for PCMG Wall Footing On Soil Using Hough Method:
Ref. LRFD Section 10.6.2.4.2, pg. 10-49

Assumptions:
B = 2 ft
Maximum grade rise is 5 feet
Soil thickness below footing is 4 feet
Use N1 of 40 (assumed corrected N_{60} value for very dense till or compacted fill)
I Influence factors from LRFD Figure 10.6.2.4.1-1, pg. 10-49
Bearing Capacity Indices (C') from LRFD Figure 10.6.2.4.2-1, pg. 10-52

$N1 := 40 \qquad I := 0.6 \qquad C' := 135$

$\sigma_o := (120pcf - 62.4pcf) \cdot 3.5ft$

$\Delta\sigma_v := 5ft \cdot 125pcf \cdot I \qquad \Delta\sigma_v = 0.38 \cdot ksf$

$$\Delta H := 4ft \cdot \left(\frac{1}{C'}\right) \cdot \log\left(\frac{\sigma_o + \Delta\sigma_v}{\sigma_o}\right)$$

$\Delta H = 0.16 \cdot in$

OK, Say 1/4 inch or less settlement below PCMG wall footing on soil. Settlement of PCMG wall on bedrock will be negligible.